

into three classes: (a) Results of observations on the 1913 flood; (b) Results of current meter measurements; (c) Miami River cut-off channel observations. The results of these observations are given in table XVIII of the Appendix.

The methods of determining the discharges during the 1913 flood, from which velocities were computed, are given in detail in another of the Technical Reports,* and will therefore not be repeated here. While evidence of scour was apparent at but a few of these bridges, it is possible that during the period of maximum velocities some scour of the bed may have occurred, which was filled again by the lower velocities during the latter part of the flood. The measurements of the second class were made with a current meter, their primary purpose being the determination of discharges at high stages of the river. The observations on the Miami River cut-off are described fully in Part IV, of the Technical Reports. There, also, velocities were determined by means of a current meter.

While the evidence is somewhat conflicting, because of the many variable factors which enter into scouring, it appears that in case of coarse gravel, like that forming the bed of the Miami River, if the bridge piers are of good design, and have reasonably deep foundations, temporary maximum velocities of 10 feet per second are safe. Where velocities higher than this were found necessary in the flood control plans, the foundations are protected by a line of steel sheet piling across the river just below the piers. The estimated maximum velocity in any of the bridge openings, for a runoff forty per cent greater than that of March 1913, is 13.2 feet per second at the High-Main Street bridge in Hamilton.

The greatest velocity which was assumed to be safe in a straight channel was also 10 feet per second. This is considerably above the velocity which caused movement in the Miami River cut-off. The conditions at that point were extremely favorable to erosion, however, and the results are hardly applicable to the conditions existing in an improved channel. Where the mean velocity will exceed 10 feet per second, the sides of the channel are paved with concrete slabs, and the bottom for some distance out from the bank is covered with a flexible concrete block revetment. Where the mean velocity exceeds 6 feet per second at bends, this protection also is placed on the outside bank.

BACKWATER CURVES

In the design of the channel improvement through the cities a large number of backwater curves were worked out to determine the

* Calculation of Flow in Open Channels, by Ivan F. Houk. Technical Reports, Part IV; The Miami Conservancy District, Dayton, Ohio, 1918.

flow line for various discharges and channel conditions. The situation at some of the cities of the District, however, particularly at Hamilton, was unusual, and necessitated the use of a slightly different form of computation from the usual one for backwater curves. In this computation the velocity head at the various sections of the stream was considered, as well as the effect of friction.

Under ordinary conditions of flow in channels, when the velocity is low and the cross section is nearly uniform, the changes of velocity head are so slight that they may be safely neglected. With high velocities, however, a slight change in cross section causes relatively a much greater change of velocity head. At Hamilton the velocities of flow in the improved channel are high and the changes of cross section considerable. To neglect the velocity head in this case would give rise to considerable error. This has been pointed out by I. P. Church in the Engineering Record of August 9, 1913, Vol. 68, page 168.

Figure 111 shows the profiles of the water surface for a flow of 200,000 second feet in the improved channel: (a) with velocity head considered; and (b) with velocity head changes neglected. In the lower part of the channel the cross section is uniform and the profiles are identical. Farther up, however, changes of cross section occur and the profiles diverge, the greatest difference (1.35 feet) being at the upper end, where water enters the improved channel. Because of the large cross section above this point, the water flows at low velocity and to increase this to the high velocity of the improved channel requires the expenditure of considerable head.

In tables 14 and 15 the computations for the two profiles are given.

While the method of computing backwater curves is familiar to most engineers, in order to furnish a guide to those who are not acquainted with the process, as well as to illustrate the difference between the two forms, the computations for both assumptions are worked out in detail.

The channel is divided into a number of sections, and the elevations of the water surface at the end of each of the sections is determined with the use of Kutter's formula, by a cut and try process, beginning with an assumed surface elevation at the lower end of the section farthest downstream. The first step is to estimate the elevation of the upper end of this section or the slope of the water surface in it. The discharge which would take place in the section is then computed, different elevations or slopes being tried until the discharge obtained is equal to the flow for which the backwater curve is desired. The computations may conveniently be arranged as in the

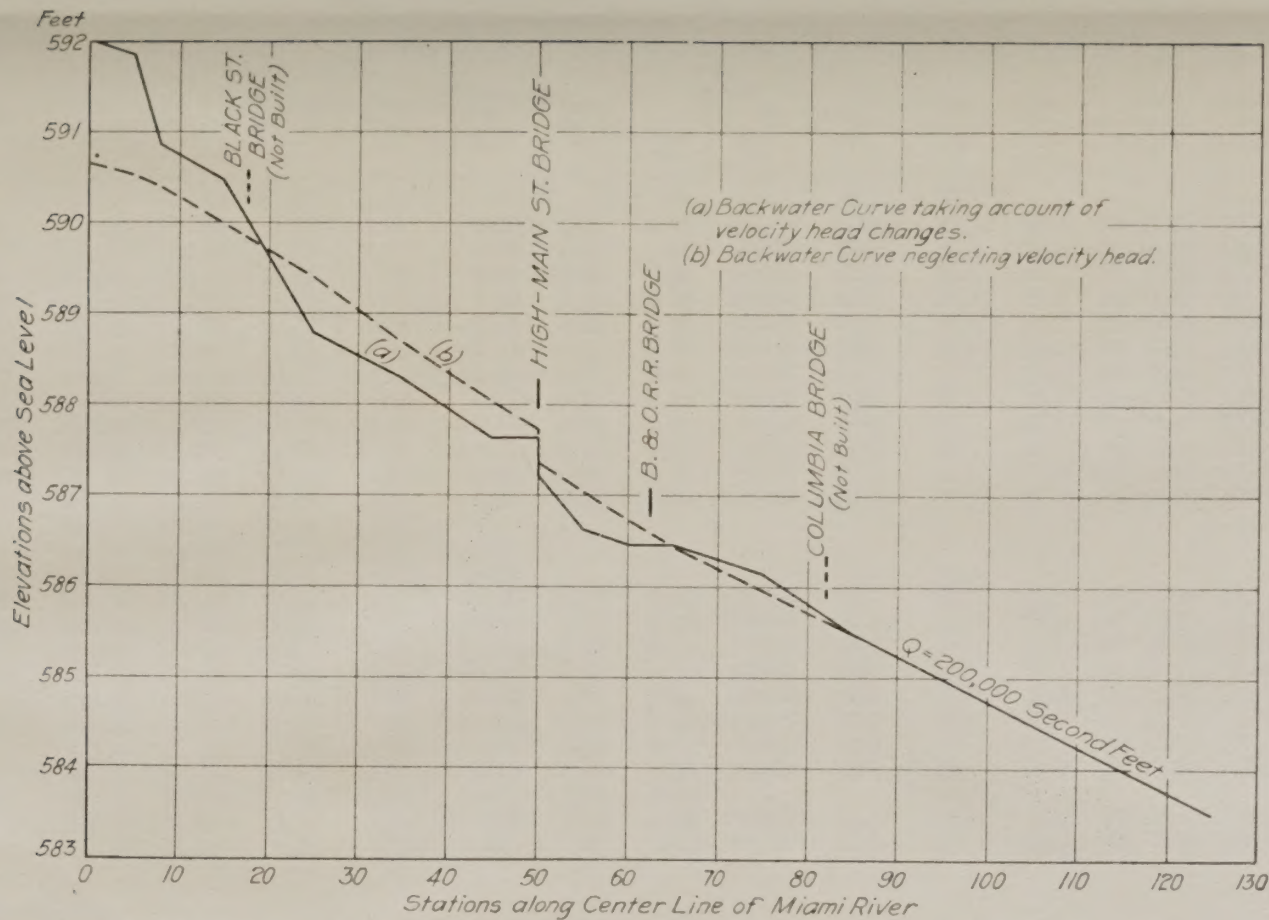


FIG. 111.—BACKWATER CURVES FOR MIAMI RIVER AT HAMILTON, OHIO.

The full line shows the computed profile taking velocity head into consideration. The dotted line shows the profile with velocity head changes neglected. Both profiles are for the improved channel assuming a discharge of 200,000 second feet. The depth of water, over 25 feet, makes it impossible to show river bottom elevations; the profile, therefore, shows water surface elevations only.

Table 14.—Computations of Backwater Curve for 200,000 Second foot Flood at Hamilton with Velocity Head Changes Considered

Sta.	Length of Section Ft.	Surface Elevation Ft.	Area, Sq. Ft.	Velocity, Ft. Per Sec.	Vel. Head, Ft.	Change Vel. Head, Ft.	Elev. Head, Ft.	Friction		Perimeter, Ft.	Hyd. Radius, Ft.	Ave. Rad., Ft.	C.	Velocity, Ft.	Average Area, Sq. Ft.	Discharge, Sec. Ft.
								Head, Ft.	Slope, Ft.							
125	—	583.50	17,750	11.27	1.97	—	—	—	—	670	26.5	—	—	—	—	—
115	1000	584.00	17,750	11.27	1.97	0.00	0.50	0.50	.00050	670	26.5	26.5	98	11.28	17,750	200,000
105	1000	584.50	17,750	11.27	1.97	0.00	0.50	0.50	.00050	670	26.5	26.5	98	11.28	17,750	200,000
95	1000	585.00	17,750	11.27	1.97	0.00	0.50	0.50	.00050	670	26.5	26.5	98	11.28	17,750	200,000
85	1000	585.50	17,750	11.27	1.97	0.00	0.50	0.50	.00050	670	26.5	26.5	98	11.28	17,750	200,000
75	1000	586.15	18,650	10.72	1.78	-0.19	0.65	0.46	.00046	700	26.8	26.7	99	10.98	18,200	200,000
65	1000	586.47	17,995	11.11	1.92	+0.14	0.32	0.46	.00046	680	26.5	26.8	99	10.99	18,320	201,000
60	500	586.47	16,070	12.45	2.41	+0.49	0.00	0.49	.00098	610	26.3	26.4	98	15.76	17,030	269,000
*55	500	586.67	15,180	13.17	2.69	+0.18	0.20	0.38	.00076	590	25.7	26.0	98	13.77	15,625	215,000
*55	500	586.60	15,220	13.15	2.69	+0.18	0.13	0.31	.00062	590	25.8	26.1	98	12.47	15,645	195,000
55	500	586.62	15,230	13.13	2.68	+0.17	0.15	0.32	.00064	590	25.8	26.1	98	12.65	15,650	198,000
*50	500	587.20	16,160	12.38	2.38	-0.30	0.58	0.28	.00056	575	28.1	27.0	98	12.05	15,700	189,200
50	500	587.25	16,185	12.35	2.37	-0.29	0.63	0.34	.00068	575	28.2	27.0	98	12.78	15,710	201,000
Bridge		587.25	15,210	13.15	2.69											
*50		587.57	16,350	12.23	2.32	+0.37										
50		587.63	16,390	12.21	2.31	+0.38				575	28.5					
45	500	587.63	15,150	13.20	2.71	+0.40	0.00	0.40	.00080	575	26.4	27.4	97	14.36	15,770	226,000
35	1000	588.30	15,060	13.27	2.74	+0.03	0.67	0.70	.00070	575	26.2	26.3	97	13.30	15,100	201,000
25	1000	588.80	14,320	13.97	3.03	+0.29	0.50	0.79	.00079	595	24.1	25.1	97	13.65	14,690	201,000
*15	1000	590.30	17,510	11.42	2.03	+1.00	1.50	0.50	.00050	660	26.5	25.3	98	11.03	15,920	175,500
15		590.47	17,620	11.35	2.00	+1.03	1.67	0.64	.00064	660	26.7	25.4	98	12.50	15,970	199,500
*8	700	591.05	17,870	11.20	1.95	-0.05	0.58	0.53	.00076	670	26.7	26.7	97	13.81	17,750	245,000
*8		590.82	17,720	11.29	1.98	-0.02	0.35	0.33	.00047	670	26.4	26.6	98	10.95	17,670	193,500
8		590.85	17,740	11.27	1.97	-0.03	0.38	0.35	.00050	670	26.5	26.6	98	11.30	17,680	200,000
*5	300	592.25	24,450	8.18	1.04	-0.93	1.35	0.42								
*5		591.90	24,170	8.27	1.06	-0.91	1.05	0.14	.00047	910	26.6	26.6	99	11.09	20,960	232,000
5		591.87	24,140	8.28	1.06	-0.91	1.02	0.11	.00037	910	26.5	26.5	99	9.80	20,940	205,000

* Trial computation.

Table 15.—Computations of Backwater Curve for 200,000 Second foot Flood at Hamilton with Velocity head changes neglected

Sta.	Length of Section, Feet	Water Surface		Area, Sq. Ft.	Wetted Perimeter, Ft.	Hydr. Radius, Ft.	Av. Hydr. Radius, Ft.	C.	Velocity Ft. per Sec.	Ave. Area, Sq. Ft.	Discharge, Sec. Ft.
		Slope	Elevation, Ft.								
125	—	—	583.50	17,750	670	26.5	—	—	—	—	—
115	1000	.00050	584.00	17,750	670	26.5	26.5	98	11.28	17,750	200,000
105	1000	.00050	584.50	17,750	670	26.5	26.5	98	11.28	17,750	200,000
95	1000	.00050	585.00	17,750	670	26.5	26.5	98	11.28	17,750	200,000
85	1000	.00050	585.50	17,750	670	26.5	26.5	98	11.28	17,750	200,000
75	1000	.00048	585.98	18,530	700	26.5	26.5	98	11.05	18,140	201,000
65	1000	.00048	586.46	17,990	680	26.5	26.5	98	11.05	18,260	202,000
60	500	.00054	586.73	16,230	610	26.6	26.6	98	11.72	17,110	201,000
*55	500	.00058	587.02	15,580	590	26.4	26.5	98	12.15	15,900	193,000
55	500	.00062	587.04	15,590	590	26.4	26.5	98	12.56	15,910	200,000
†50	500	.00060	587.34	16,230	575	28.2	27.3	98	12.55	15,910	200,000
Under bridge	—	—	587.34	15,190	—	—	—	—	13.17	—	200,000
Above bridge	—	—	587.73	16,430	575	28.6	—	—	12.17	—	200,000
45	500	.00060	588.03	15,360	575	26.7	27.7	98	12.64	15,900	201,000
*35	1000	.00062	588.65	15,250	575	26.5	26.6	98	12.60	15,300	193,000
35	1000	.00066	588.69	15,270	575	26.6	26.6	98	12.98	15,310	199,000
25	1000	.00071	589.40	14,650	595	24.6	25.6	98	13.21	14,960	198,000
*15	1000	.00060	590.00	17,320	660	26.2	25.4	98	12.10	15,980	193,000
15	1000	.00064	590.04	17,340	660	26.3	25.4	98	12.50	15,990	200,000
8	700	.00052	590.40	17,440	670	26.0	26.1	98	11.42	17,390	199,000
*5	300	.00036	590.51	22,980	910	25.2	25.6	99	9.48	20,210	191,000
5	300	.00039	590.52	22,990	910	25.2	25.6	99	9.88	20,210	200,000

* Trial computation.

† Immediately below High-Main St. bridge

examples given, the sequence of the steps being the same as the arrangement of columns from left to right. Some error probably will be made in the estimate of the surface elevation at the lower end of the channel, but as the computations are carried further upstream the effect of this error becomes less and less. To secure an accurate profile, it is therefore necessary to begin some distance below that portion of the channel for which accuracy is desired. For the profiles at Hamilton, the elevation assumed at station 125 was determined by previous calculations.

In the computations given, the hydraulic radius of the section was assumed to be the mean of the hydraulic radii of the two end cross sections. Another assumption which would give possibly as good results is that the hydraulic radius for the section is equal to the mean of the two end areas divided by the mean of the wetted perimeters of the end sections. The number of steps in the computation is the same for either of these assumptions.

In considering the changes of velocity head it was assumed that in no case could the decrease in velocity head cause the surface to slope upward in the direction of flow. In sections 65-60 and 50-45, therefore, where the change in velocity head is more than sufficient to overcome friction, the surface slope was assumed to be zero.

The method of computing the drop due to the effect of the bridge is also illustrated. This is based on the assumption that the surface, as the water enters the bridge openings, drops to the elevation of that just below the bridge, and the heading up which occurs is just sufficient to increase the velocity of the water approaching the bridge to the higher velocity which exists under the bridge, and to overcome the friction through the bridge. None of this head is assumed to be recovered as the velocity of the water is again reduced in the section below the bridge. It is believed that this method of computation gives results which are amply conservative.

